CHAPTER 8

VERIFICATIONS WITH FIELD LOAD TESTS

This chapter presents a number of example problems that reflect real applications for existing test projects. The SW model program input and output data will be summarized in tables. The data input will include the following sets of shaft and soil properties, shaft group geometry, and loads (shear and axial forces, and moment) applied at the top of the shaft.

8.1 INPUT DATA

8.1.1 Shaft Properties

- Shaft-head conditions (free head, fixed head, zero rotation or zero deflection)
- Behavior of shaft material (linear or nonlinear analysis)
- Shaft-head location above or below ground surface
- Shaft length
- Number of shaft segments (≥ 1)
- Length of shaft segment
- Diameter of shaft segment
- Uniaxial strength of concrete after 28 days (f_c)
- Longitudinal steel ratio(s) (A_s/A_c)
- Steel yield stress
- Thickness of steel casing, if present
- Steel yield stress of steel casing
- Thickness of concrete cover

Based on the ACI formula, the bending stiffness (EI) of the shaft cross-section is determined internally by the program S-Shaft.

8.1.2 Soil Properties

- Number of soil layers (starting from ground surface)

- Uniform surcharge at the ground surface (additional uniform loads at ground surface)
- Location of water table below ground surface
- Soil type of each soil layer
- Thickness of soil layer
- Effective unit weight of soil (γ)
- Friction angle (φ) for sand
- Undrained shear strength for clay (S_u)
- Unconfined compressive strength of rock mass (q_u)
- ε_{50}** of sand, clay, C-φ soil or rock (charts provided and can be determined by the program for default =0)

8.1.3 Liquefaction Analysis (for Saturated Sand)

- Corrected number of blowcounts, $(N_1)_{60}$
- Percentage of fines in sand
- Shape parameter (roundness) of sand grains

** ε_{50} = axial strain of soil at 50% of stress level (i.e. 50% of soil strength). ε_{50} can be calculated internally in the shaft program by typing 0 (program default). Also, a chart is provided in the program help to allow the designer to check the values of ε_{50}. However, it is recommended that the chart be used for sand if the uniformity coefficient (C_u) > 2 (from sieve analysis data). For the case of rock mass, the curve of S_u vs. ε_{50} is extrapolated to cover the rock mass strength ($q_u = 2\ S_u$).

8.1.4 **Loads** (shear force, moment and axial load)

- Axial load at shaft head
- Bending moment at shaft head
- Desired lateral load (shear force) at shaft head

8.1.5 Earthquake Excitation (Liquefaction)

- Magnitude of earthquake (M)
- Peak ground acceleration (a_{max})

8.2 LAS VEGAS FIELD TEST (SHORT SHAFT)

The Las Vegas test for large 8-foot diameter shaft represents an excellent case study for a short shaft (Zafir and Vanderpool, 1998). The soil data input for use with the programs FLPIER/COM624 was evaluated by the University of Florida team. The same soil data input has been used in the SW model program. The nonlinear modeling of shaft material (concrete and steel) is employed in both the FLPIER/COM624 and SW Model analyses.

The reinforced concrete drilled shaft tested was 8 feet in diameter and 32 feet in length with 1% longitudinal steel reinforcement. The uniaxial strength of concrete after 28 days (f_c) is assumed to be 5.0 ksi. Table 8-1 summarizes the detailed information for the soil profile as reported by the University of Florida team.

Table 8-1 - Soil Profile for the Las Vegas Test

Soil layer	Soil type	Thickness (ft)	γ (pcf)	ϕ (deg.)	k (pci)
Layer 1	Sand	2.5	120	33	15
Layer 2	Sand	6.5	120	37	30
Layer 3	Sand	3.0	120	32	11
Layer 4	Sand	1.5	120	36	26
Layer 5	Sand	7.5	120	45	62
Layer 6	Sand	2.0	120	40	43
Layer 7	Sand	3.5	120	45	63
Layer 8	Sand	6.0	120	40	44
Layer 9	Sand	1.0	120	32	10
Layer 10	Sand	2.0	120	37	32

γ = effective unit weight of soil

k = coefficient of subgrade reaction (F/L^3)

Compared to COM624/FLPIER, the SW model program provides very good prediction for the laterally loaded large diameter short shaft of the Las Vegas test (see Table 8-2 and Figs. 8-1 through 8-4). The nonlinear modeling of shaft material is used to show the program capability of predicting the response of a large diameter short shaft.

Table 8-2 Comparison of Measured Shaft Head Deflection and SW model and FLPIER/COM624P Predictions for Las Vegas Test

Load (kips)	Actual Shaft-Head Deflection, Yo, in	SW Model Deflection, Yo, in	FLPIER/COM624 Deflection, Yo, in
50	0.02	0.02	0.201
100	0.04	.05	0.402
150	0.07	.08	0.603
200	0.125	0.11	0.804
300	0.235	0.22	1.27
400	0.40	0.38	1.89
500	0.61	0.58	2.76
600	0.88	0.88	3.9
700	1.21	1.25	5.75
750	1.36	1.45	7.15

8.3 SOUTHERN CALIFORNIA FIELD TEST (SHORT SHAFT)

A number of cast-in-place drilled piers were constructed and tested in Southern California and reported by Bhushan et al. (1978). The piers were constructed at five different sites. One of these piers is an ideal short shaft with which to compare the predictive capability and reliability of computer programs such as LPILE/COM624P/FLPIER and SW model. Regardless of the predicted results, it should be mentioned again that the p-y curves employed in the programs LPILE, COM624P or FLPIER were established based on long small diameter piles that are not representative of large diameter shafts.

In short shaft case reported here, the pier tested was 4 feet in diameter and 16 feet in length. The pier was constructed in stiff clay with undrained shear strength (S_u) of 5500 psf and ε_{50} of 0.94% (Table 8-3). This data was reported by Bhushan et al. (1978) and used with COM624 by Reese (1983) [the developer of the program COM624 and LPILE]. Reese (1983) reported the results provided by the program COM624 and presented in Fig. 8-5 and Table 8-4. Compared to the measured data, COM624 provides very soft response. The results assessed using the SW model program are in good agreement with the field data. Figures 8-6 through 8-8 show the lateral response of the tested shaft using the SW model technique.

Table 8-3 - Soil Data for Southern California Test

Soil Layer	Soil Type	Thickness (ft)	γ (pcf)	ϕ (deg.)	S_u (psf)	ε_{50}**
Layer 1	Clay	22	130	34	5500	0.0095

Table 8-4 Comparison of Measured Shaft Head Deflection and SW model and COM624P Predictions for Southern California Test

Load (kips)	Actual shaft-head deflection, Yo, in	SW model Deflection, Yo, in	COM624 Deflection, Yo, in
50	0.1	0.094	0.20
100	0.25	0.2275	0.35
200	0.67	0.59	1.50
300	1.10	1.00	4.40
400	1.85	1.55	15.0

8.4 TREASURE ISLAND FULL-SCALE LOAD TEST ON PILE IN LIQUEFIED SOIL

A series of full-scale field tests in liquefied soil was performed at Treasure Island in San Francisco Bay (Ashford and Rollins 1999). The soil properties employed in the SWM analysis for the test site based on the reported data (Weaver et al. 2001) are described in Table 8-5. Soil and pile properties can be also seen in Fig. 8-9. In this analysis, the sand is assumed to contain 5% fines. The soil was liquefied by carrying out controlled blasts at that site without densifying the soil in the test area. Drained and undrained lateral loading tests were performed on a long isolated pipe pile filled with concrete (CISS) of 0.61 m diameter. The tested pile exhibited free-head conditions and was laterally loaded 1.0 m above ground surface. The test pile had bending stiffness EI = 448320 kN-m^2.

The observed and SW model predicted drained response of the pile compares favorably as seen in Fig. 8-10. Procedures followed in the Treasure Island test (liquefying the soil around the pile and then loading the pile laterally) subsequent to the static drained test are similar to those employed with the SW model analysis. The assessed undrained post-liquefaction behavior of the tested pile is based on the procedures presented herein, and includes the effect of free-field and near-field excess pore pressure ($u_{xs,ff}$ + $u_{xs,nf}$). The pile head response shown in Fig. 8-10 is based on a peak ground acceleration (a_{max}) of 0.11g, and an earthquake magnitude (M) of 6.5.

The piles were cyclically loaded after the first blast at the site. The observed (field) undrained points (Ashford and Rollins 1999), which are shown in Fig. 8-10, represent the peaks of the cyclic undrained response of these piles. It should be mentioned that the good agreement between the measured and predicted undrained response is based on an assumed maximum ground acceleration, a_{max}, of 0.11g. This value of a_{max} generates high excess porewater pressures ($u_{xs,\ ff}$) in most of the sand layers. It should be noted that the value of a_{max} employed in the analysis causes an excess porewater pressure ratio (r_u) equal to 0.95 in most of the sand and the best match with the measured free-field excess porewater pressure pattern induced in the field (Ashford and Rollins 1999).

Table 8-5. Soil Properties Employed in the SWM Analysis for the Treasure Island Test

Soil Layer Thick. (m)	Soil Type	Unit Weight, $\bar{\gamma}$ (kN/m^3)	$(N_1)_{60}$	ö (degree)	å$_{50}$ %	S_u KN/m 2
0.5	Brown, loose sand (SP)	18.0	16	33	0.45	
4.0	Brown, loose sand (SP)	8.0	11	31	0.6	
3.7	Gray clay (CL)	7.0	4		1.5	20
4.5	Gray, loose sand (SP)	7.0	5	28	1.0	
5.5	Gray clay (CL)	7.0	4		1.5	20

The p-y curve comparisons in Fig. 8-11 show the capability of the SW model for predicting the p-y curves of a pile/shaft in fully or partially liquefied soils. The back-calculated (measured) p-y curves at different depths for the 0.61-m cast-in steel-shell (CISS) pile are from Weaver et al. (2001). Other techniques, such as the traditional p-y curve approach with a reduction multiplier, do not show the concave-upward pattern of the back-calculated p-y curves.

It was obvious from the $u_{xs,\ ff}$ distribution measured along the depth of the pile right after the blast that the upper 4.6 m was almost fully liquefied. The back-calculated (field) p-y curves shown in Fig. 8-11 were assessed after a few cycles of loading. As a result, the porewater pressure in the upper 4.5 m of soil reached 1.0. By increasing the peak ground acceleration (a_{max}) used in the SW model analysis from 0.11g to 0.15g, the whole soil profile completely liquefies and the pile head response (load-deflection curve) follows the concave-up shape (increasing slope) as seen in Fig. 8-12.

8.5 COOPER RIVER BRIDGE TEST AT THE MOUNT PLEASANT SITE, SOUTH CAROLINA SITE

Cyclic lateral load tests were performed on two large diameter long shafts at the Mount Pleasant site. Shaft MP-1 (Cast-in-Steel-Shell, CISS, bending stiffness (EI) = 2 x 10^8 kip-ft^2) was 8.33 ft in diameter while the shaft MP-2 (Cast-in-Drilled Hole, CIDH, EI = 1.38 x 10^8 kip-ft^2) was 8.5 ft in diameter, each with a one-inch thick steel shell. The lateral load in both cases was applied at a point 43-inches above the ground surface. The Mount Pleasant site soil profile consists of 40 ft of loose to medium dense, clean or silty or clayey sands overlaying a thick layer of the Cooper Marl (S & ME 2000). Table 8-6 summarizes the basic soil properties of the soil profile at the Mount Pleasant site used in the SW model analysis. Lateral static load tests were carried out on as-is conditions, and liquefied conditions induced by controlled blasting (Figs. 8-13 and 14) (S & ME). The blast successfully generated high porewater pressure (r_u =1) within most of the upper 38 ft as indicated by the piezometer data.

LPILE analyses for the load test for the project were carried out using (1) the traditional p-y curve for the 38 feet thick overburden consisting of sandy deposits of $\varphi = 35^o$ and $\gamma = 60$ pcf and (2) back calculated p-y curves for the Cooper Marl from O-cell tests as no traditional p-y curves representative of the Cooper Marl conditions were available. The LPILE results for pre- and post-liquefaction conditions based on the back-calculated p-y curve from the O-cell tests are shown in Figs. 8-13 and 14. In contrast, SW model predicted p-y curves for the Cooper Marl showed good agreement with the back-calculated p-y curve from the O-cell tests. The SW model results shown in Figs. 8-13 and 14 are based on the p-y curves predicted from the SW model analysis.

Table 8-6 Soil Properties Employed in the SW Model Analysis
for the Cooper River Bridge Tests at Mt. Pleasant

Soil Layer Thick. (ft)	Soil Type	Unit Weight, $\bar{\gamma}$ (pcf)	$(N_1)_{60}$	Ö (degree)	å$_{50}$ %	S_u psf
4	Slightly clay sand (SP-SC)	120	19	34	0.004	
9	Sandy clay (CH)	62	7	30	0.008	
16	Very clayey sand (SC-CL)	62	10	32	0.006	
9	Silty sand (SM)	62	7	30	0.008	
80	Cooper Marl	65	20		0.002	4300

LPILE shaft responses for liquefied conditions were computed for various trial values of r_u different from the measured value in order to come up with a reasonable agreement of shaft response with the field test results. A constant value for r_u =0.7 for the upper 38 ft of overburden used in the LPILE analysis (for shaft MP-1) yields reasonable agreement with the field results Fig. 8-13). It should be noted that (1) r_u measured in the field was very close or equal to one and (2) use of r_u in the LPILE analysis only reduces the buoyant (effective) unit weight of soil thereby producing a softer shaft responses. r_u used with shaft MP-2 in LPILE analysis was not defined in the report (SM&E 2000).

The SW model analysis for a shaft in liquefied soil depends on several factors: earthquake magnitude (M); peak ground acceleration (a_{max}); and the soil properties to determine the values of r_u and the additional excess porewater pressure resulting from the superstructure lateral loading. An earthquake magnitude of 6.5 and a_{max} of 0.1g and 0.3g were used in the SW model analysis to obtain the shaft responses shown in Figs. 8-13 and 14. It should be noted that a_{max} of 0.3g develops complete liquefaction in the upper 38 ft of soil. Despite the diameter and EI of shaft MP-1, larger than those of shaft MP-2, shaft MP-2 experienced a post-liquefaction lateral response stiffer than that of shaft MP-1, as observed in the field test (Figs. 8-13 and 14). The use of different values of a_{max} in the SWM analysis is to exhibit the varying shaft response. Knowing the seismic zone (i.e. M and a_{max}) and soil and shaft properties at a particular site, the designer will be able to assess the lateral response of a shaft/pile in liquefiable soils using the SW model computer program. No attempt was made by SM&E 2000 to back calculate the p-y curves.

8.6 UNIVERSITY OF CALIFORNIA, LOS ANGELES (UCLA) FULL-SCALE LOAD TEST ON LARGE DIAMETER SHAFT

A full-scale load test funded by Caltrans on a cast-in-drilled-hole (CIDH) shaft/column was conducted by UCLA (Janoyan et al. 2001). The 88-ft long shaft/column tested was 6.0 ft in diameter for the 40 ft above ground and 6.5 ft in diameter for the 48 ft below ground surface. The configuration of the tested shaft and its material (concrete/steel) properties are shown in Fig. 8-15. The testing was performed at a site with deep alluvial soils consisting of silty clay and silty, clayey sand. The soil properties employed in the SW model analysis are reported in Table. 8-7. The shaft/column tested was pushed laterally up to failure (the formation of a plastic hinge). It should be noted that the field results indicate that the shaft responded as an intermediate shaft which is consistent with the SW model program description.

Figure 8-16 provides a comparison between the experimental and computed moment curvature response for the 6-ft-diameter shaft cross section. Compared to the results of the X-Section program (used by Caltrans), the moment-curvature relationship assessed using the SW model program shows better agreement with the experimental results.

Table 8-7 - Soil Data for the UCLA Test

Soil layer	Soil type	Thickness (ft)	γ (pcf)	S_u (psf)	ε_{50}
Layer 1	Stiff Clay	6	130	4000	0.003
Layer 2	Stiff Clay	18	130	2500	0.005
Layer 3	Stiff Clay	40	130	3000	0.004

Figure 8-17 shows a comparison between the measured shaft response and the computed one using LPILE, SWM6.0 and the current shaft program. To obtain good match with field data, a sand soil profile was used with LPILE (as reported by Caltrans). The data obtained using LPILE based on the original soil profile shown in Table 8-7 and given in the UCLA report did not provide good agreement with the measured data. As seen in Fig. 8-16, the same column/shaft was previously analyzed using the older SW model program (SWM6.0) for a long piles/shaft that does not account for the vertical side shear resistance and shaft classification.

8.7 FULL-SCALE LOAD TEST ON A BORED PILE IN LAYERED SAND AND CLAY SOIL

A bored 1.5-m-diameter reinforced concrete pile was installed to a depth 34 m below ground surface in the town of Chaiyi in the west central coastal plain of Taiwan (Fig. 8-18 by Brown et al. 2001). The pile tested with free-head conditions was laterally loaded at 0.5 m above ground. As reported by Brown et al (2001), relatively poor comparisons with the measured results were obtained using the traditional p-y curves for sand (Reese et al. 1974) and clay (Matlock 1970) with the program FLPIER (McVay et al. 1996). The traditional p-y curves were modified to a very large extent in the upper 12 m (see the modified p-y curves by Brown et al. 2001 in Fig. 8-19) in order to obtain good agreement with the measured data for the isolated pile.

Using the original soil properties given by Brown et al. (2001) shown in Fig. 8-18, the SW model provides an assessed response in good agreement with the measured response of the single free head pile (Fig. 8-20). A nonlinear model for pile material behavior (reinforced concrete) incorporated in the SW model analysis is employed in this analysis. It should be noted that none of the given (original) soil and pile properties was modified for the SW model analysis.

As presented by Brown et al. (2001) FLPIER (McVay et al. 1996) provides excellent agreement with the measured response by using deduced (adjusted) (site specific) modified p-y curves shown in Fig. 8-19. The nonlinear modeling of pile material played an important role in the results obtained by FLPIER and the SW model analyses. Significant recommendations and comments were made by Brown et al. (2001) relative to a p-multiplier to be used with the traditional p-y curves.

8.8 SUMMARY

This chapter has demonstrated via comparison with field test results, the capability of the SW model shaft analysis relative to various applications. The SW model analyses were undertaken using the unmodified soil and pile properties reported in the literature. Comparable LPILE/COM24/FLPIER assessments using the traditional p-y curves required moderate to significant modifications of such data in order to obtain reasonable agreement with overall field test results. Even so, traditional p-y curves for liquefied sand do not show the concave-upward shape that is predicted by the SW model analysis and noted back-calculated curves from the Treasure Island tests.

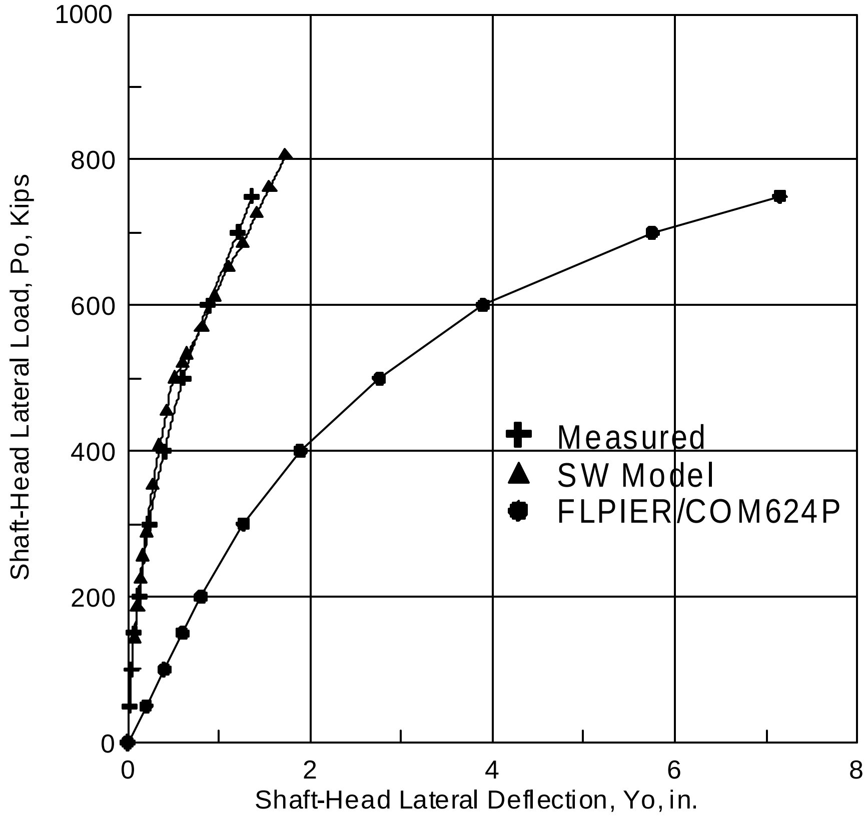

Fig. 8-1 Measured and Computed Shaft Response of the Las Vegas Test

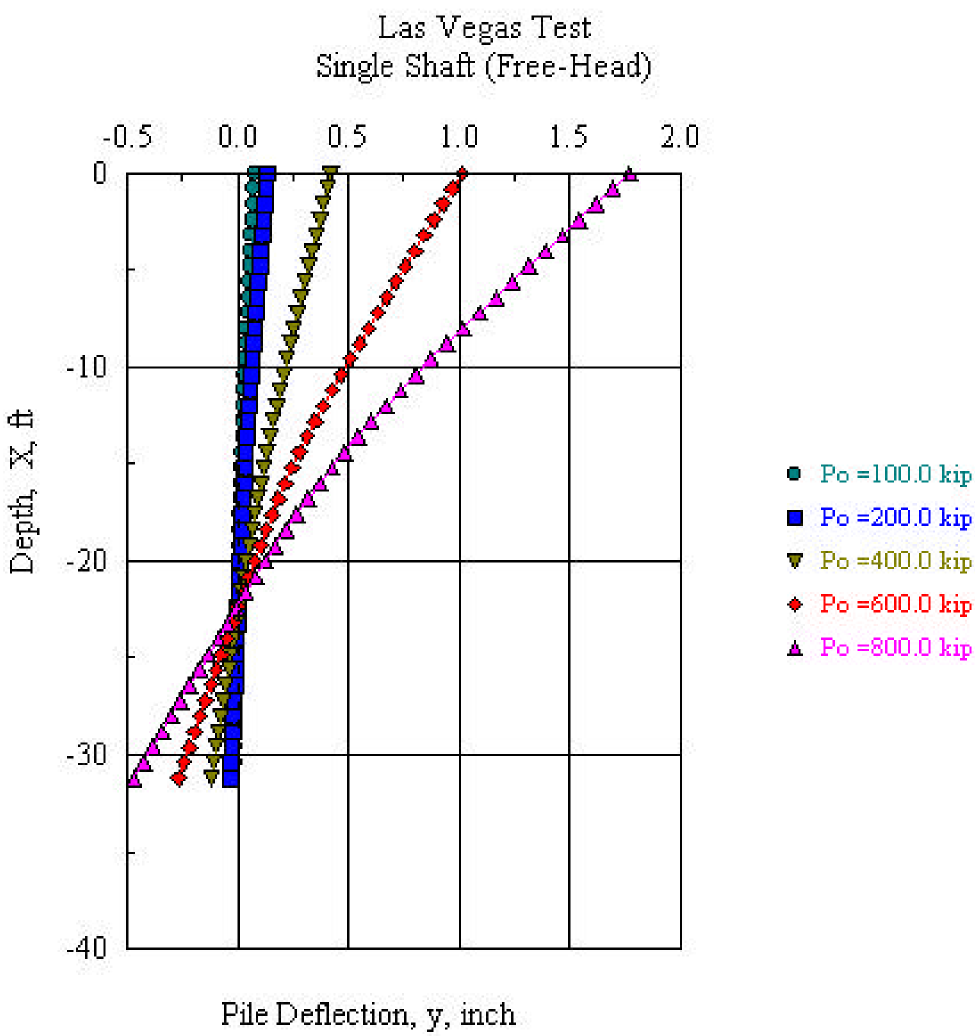

Fig. 8-2 Computed Lateral Deflection of the Shaft at Various Loads in the Las Vegas Test

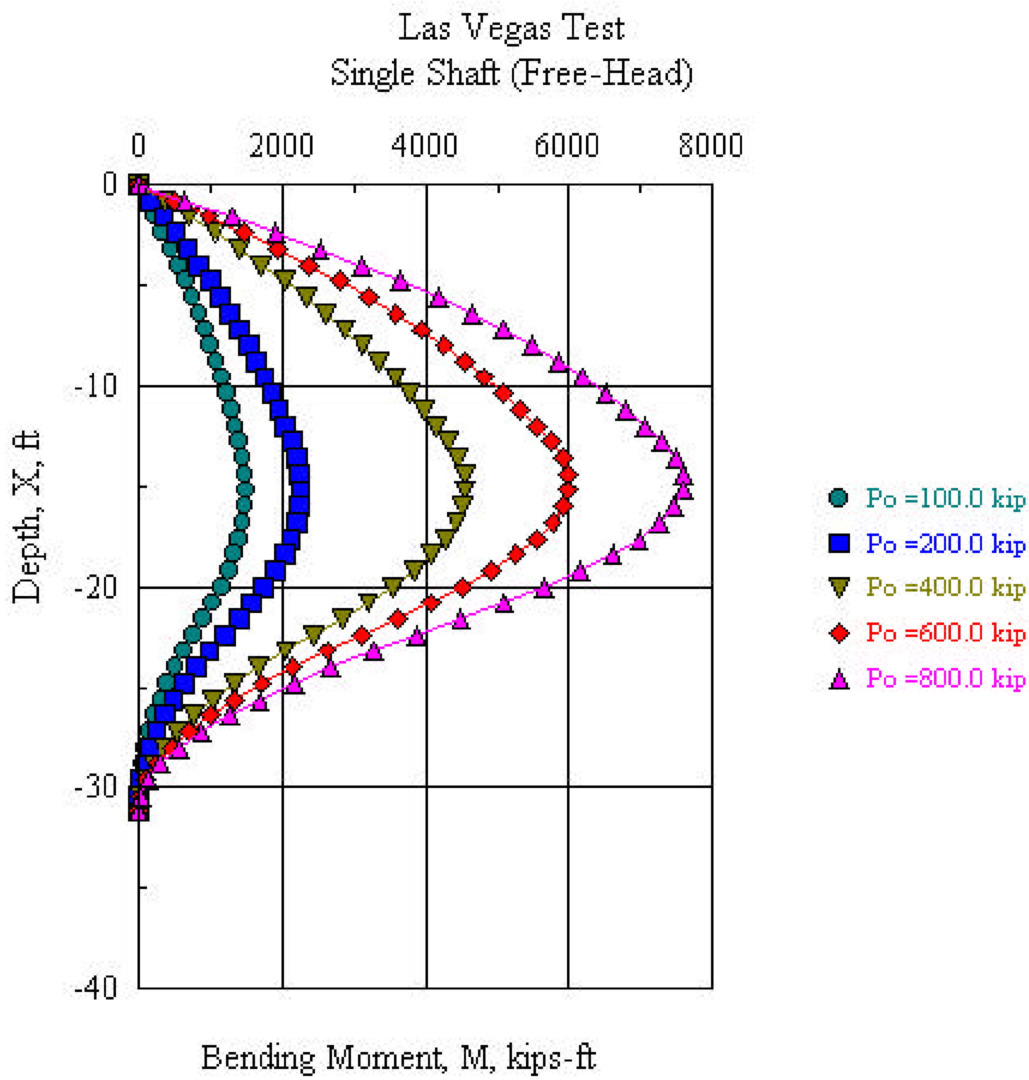

Fig. 8-3 Computed Bending Moment Distribution in the Shaft in the Las Vegas Test

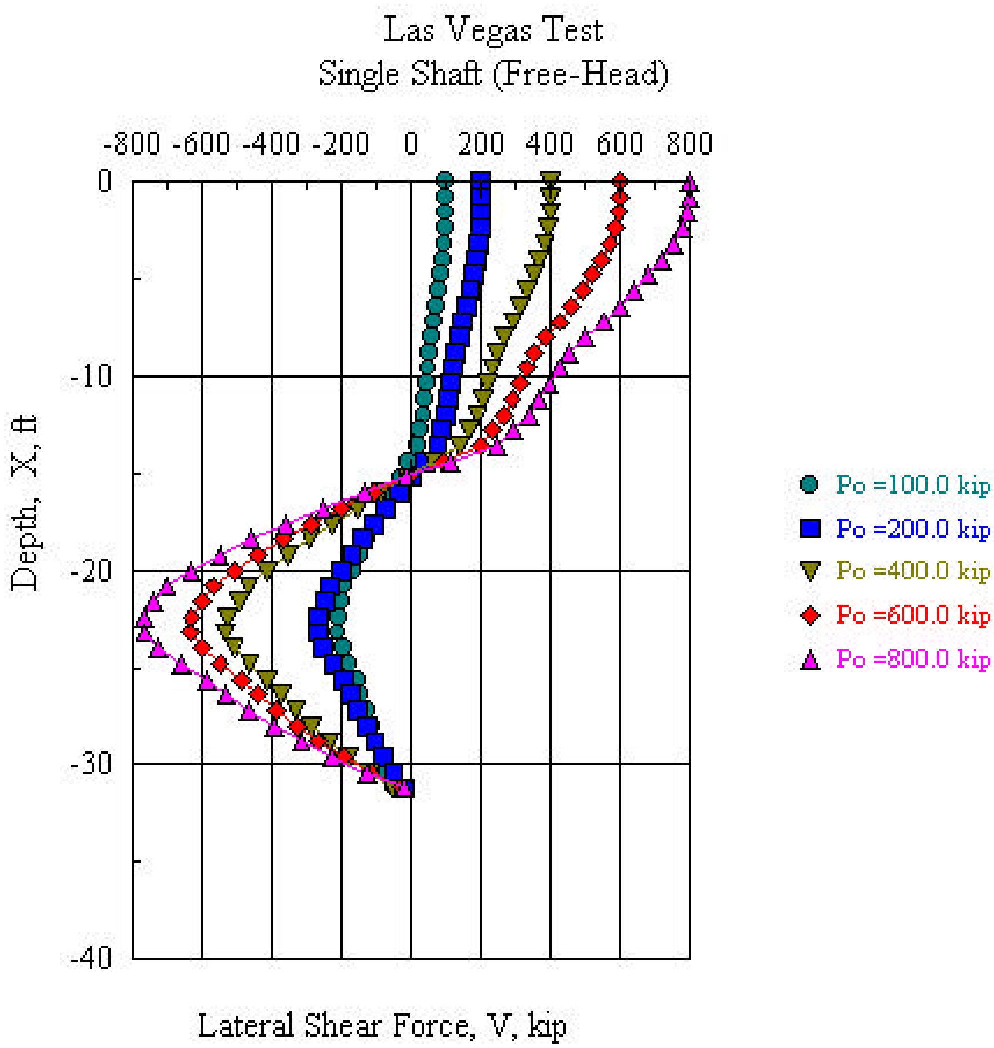

Fig. 8-4 Computed Shear Force Distribution in the shaft in the Las Vegas Test

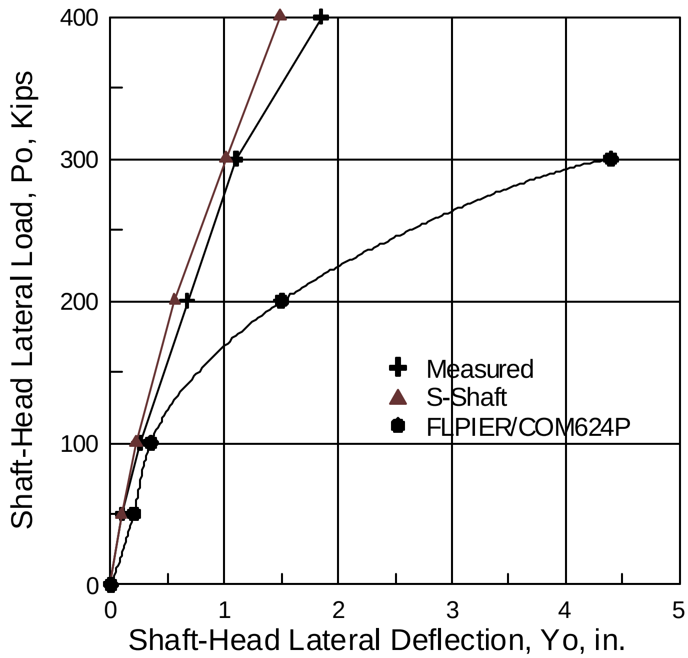

Fig. 8-5 Measured and Predicted Shaft Response of the Southern California Test (Pier 1)

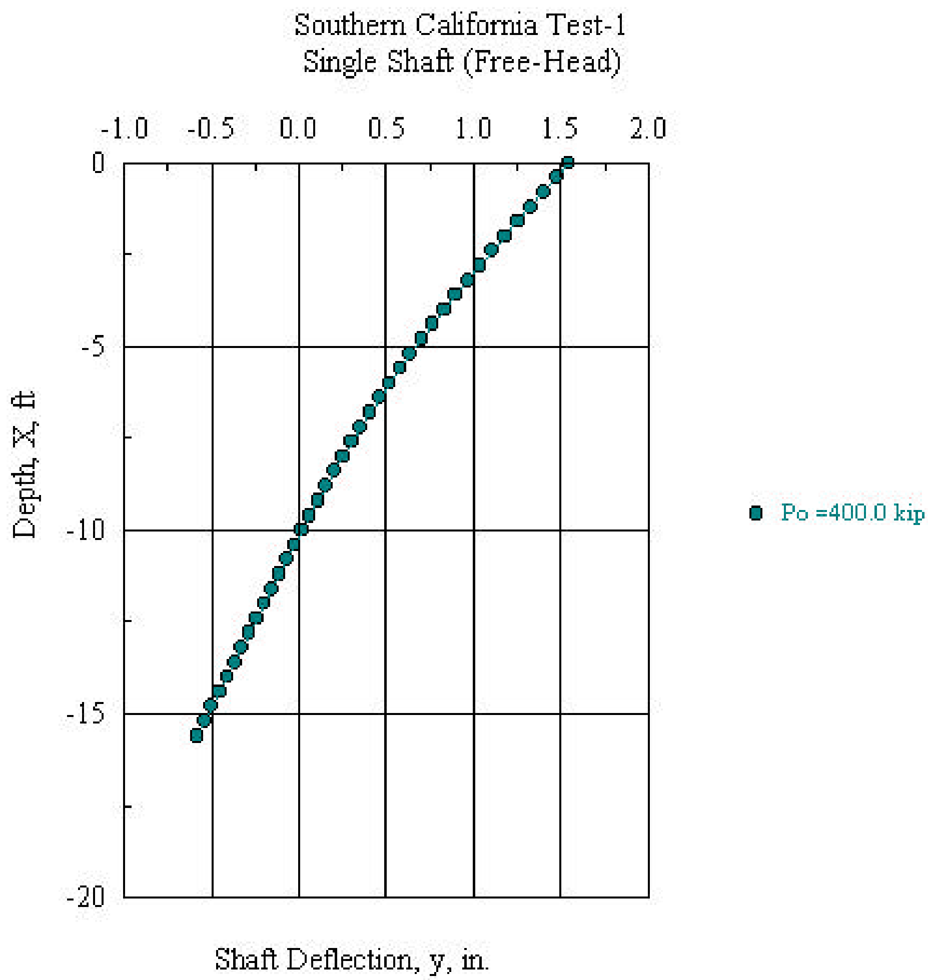

Fig. 8-6 Computed Lateral Deflection for at the Southern California Test

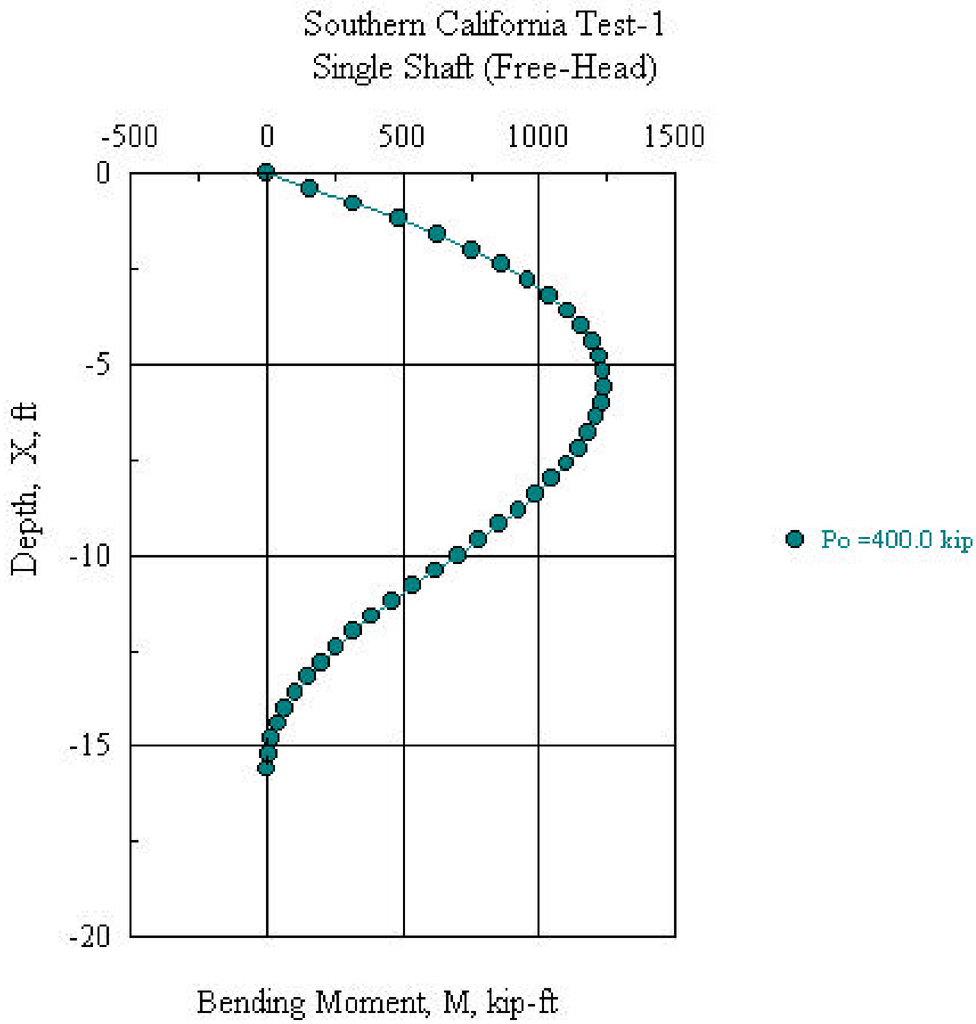

Fig. 8-7 Computed Bending Moment Distribution for the Southern California Test

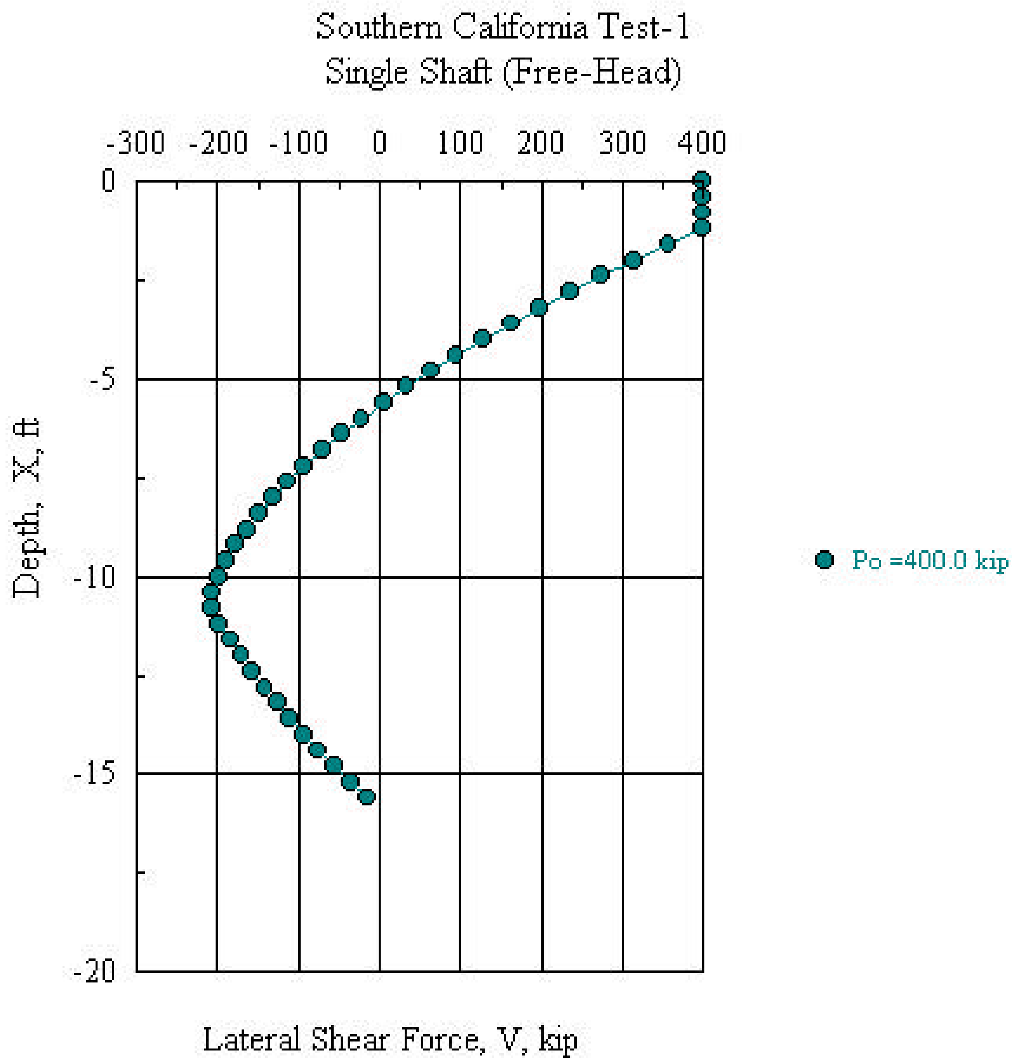

Fig. 8-8 Computed Shear Force in the Shaft in the Southern California Test

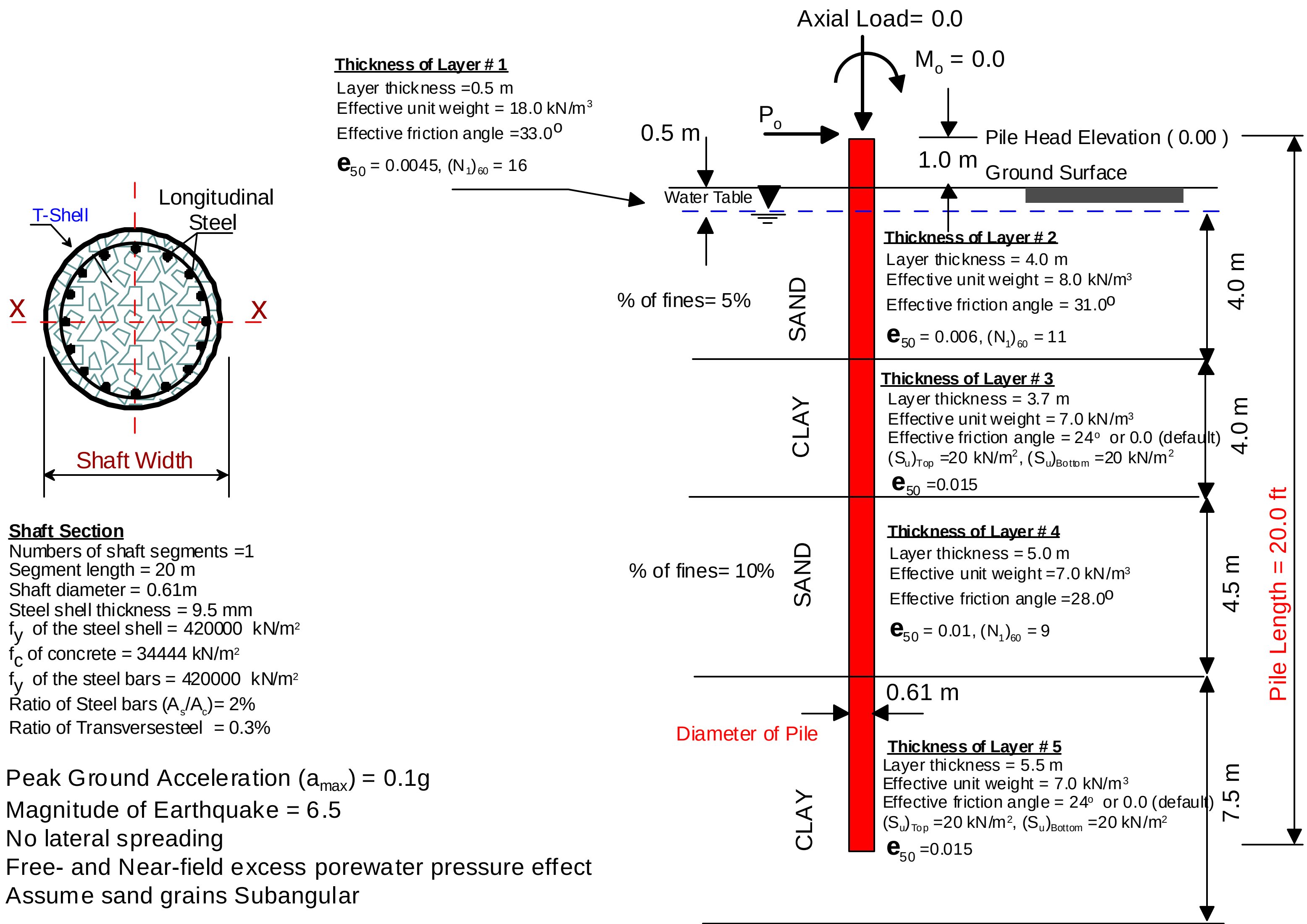

Fig. 8-9 Soil and Pile Properties of the Test Performed at Treasure Island.

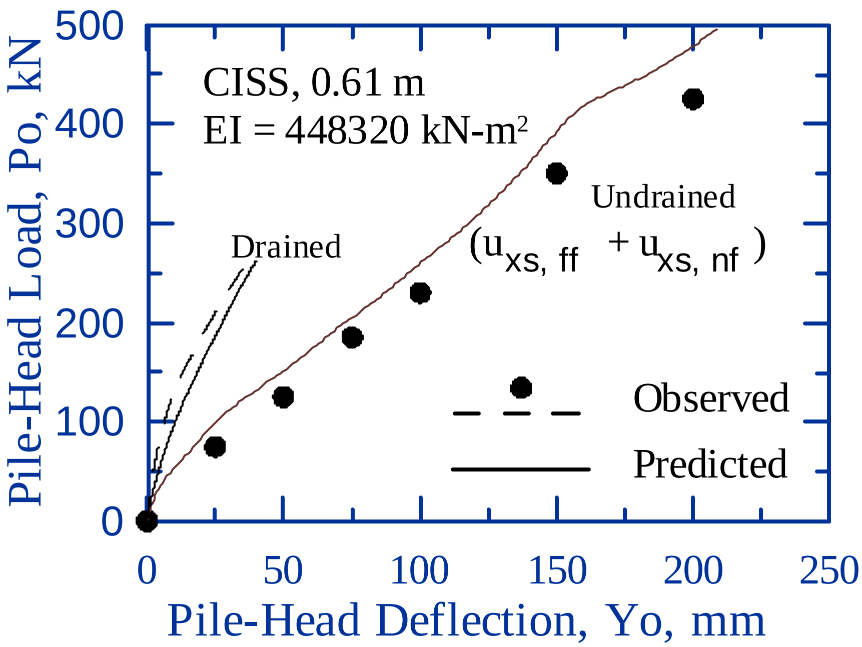

Fig. 8-10 Post-Liquefaction Pile Head Response of the Treasure Island Test

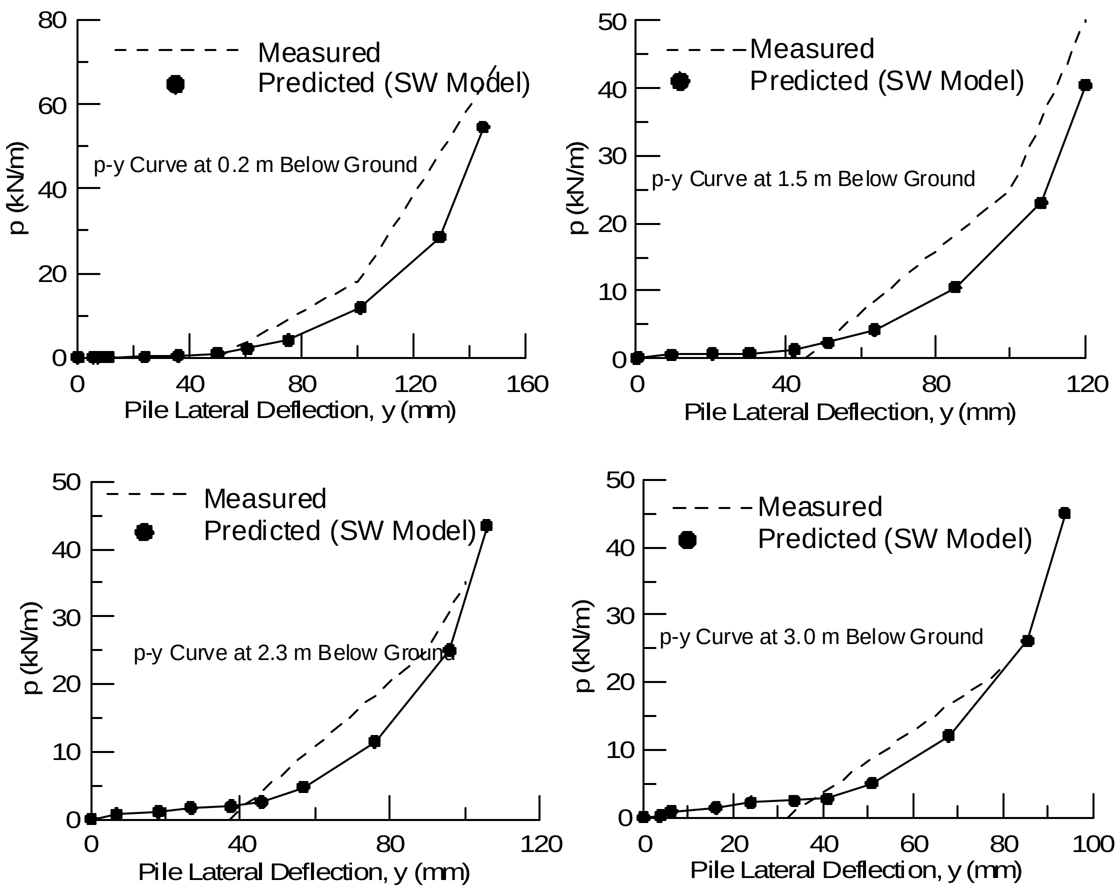

Fig. 8-11. Predicted p-y Curves Using the SWM vs. the Observed Ones from Treasure Island Test.

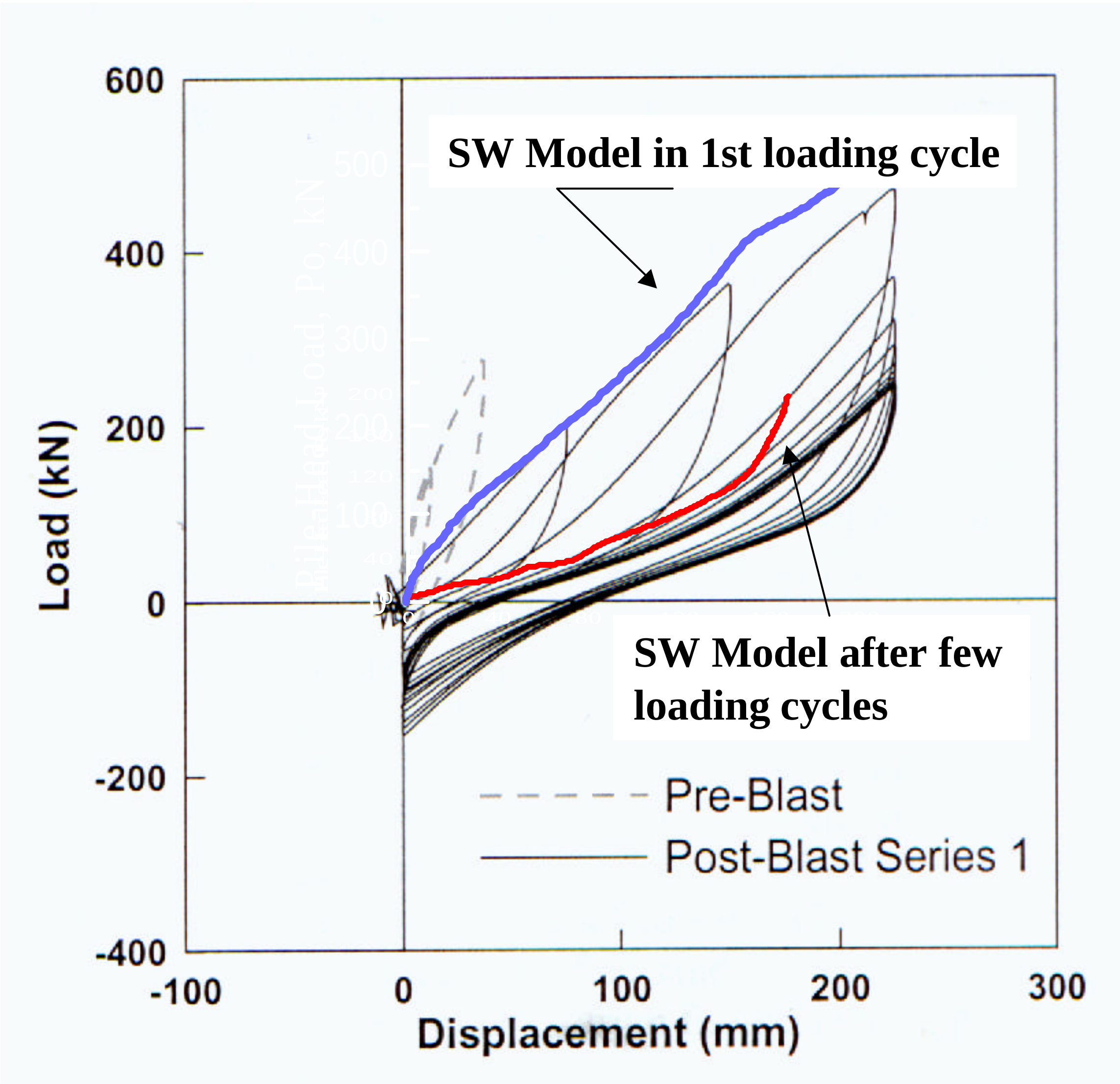

Fig. 8-12. Effect of Loading Cycles on Pile Head Response after Liquefying the Upper Soil Layers (0.61 m Diameter)

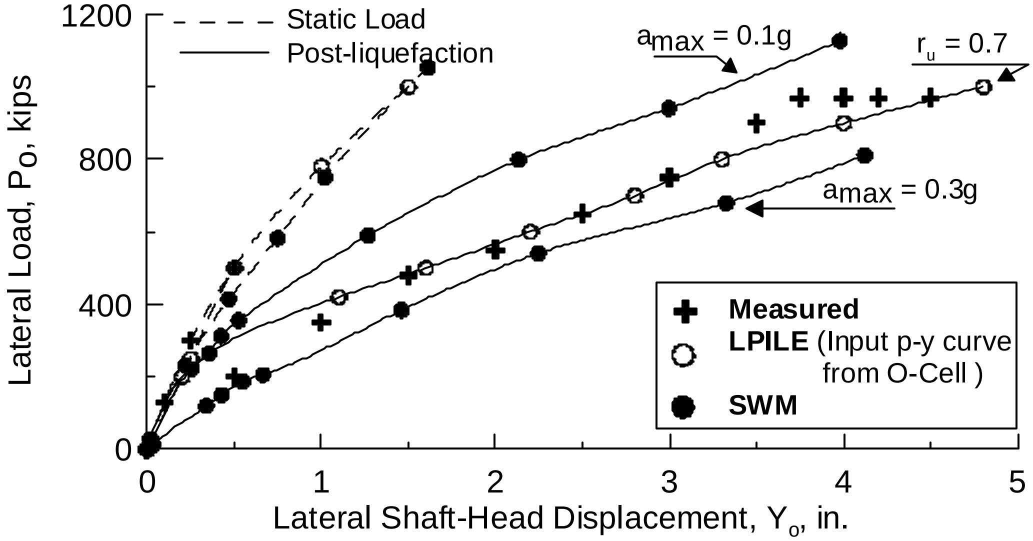

Fig. 8-13. Lateral Response of Shaft MP-1 at Mount Pleasure Test Site (Cooper River Bridge)

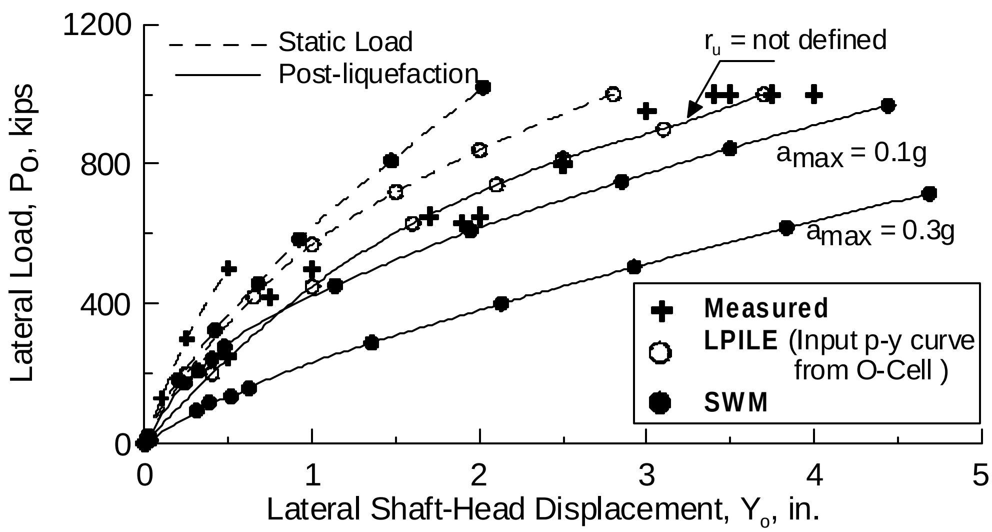

Fig. 8-14 Lateral Response of Shaft MP-2 at Mount Pleasure Test Site (Cooper River Bridge)

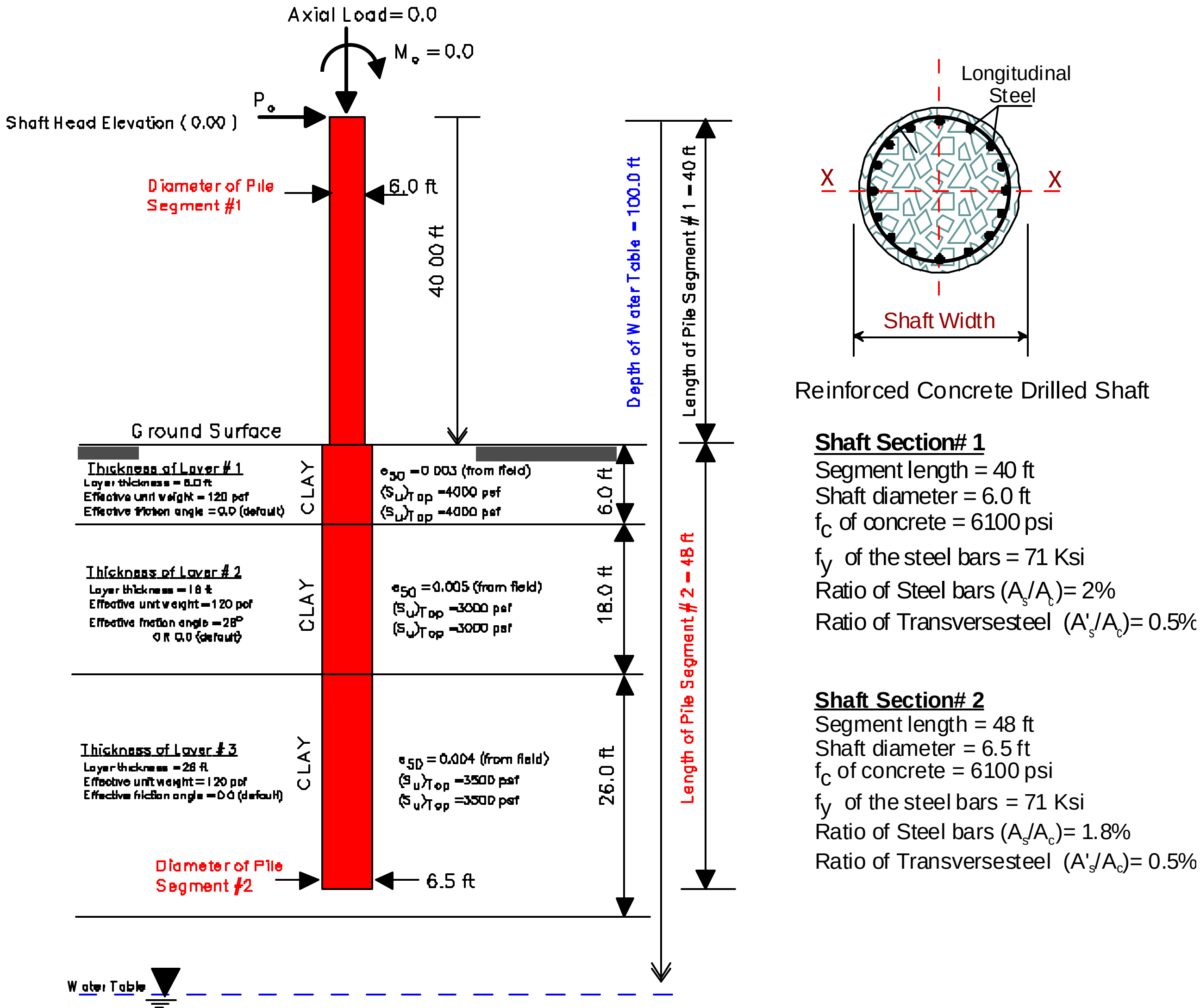

Fig. 8-15 Soil and Shaft Properties of the Column/Shaft Test Performed by the UCLA

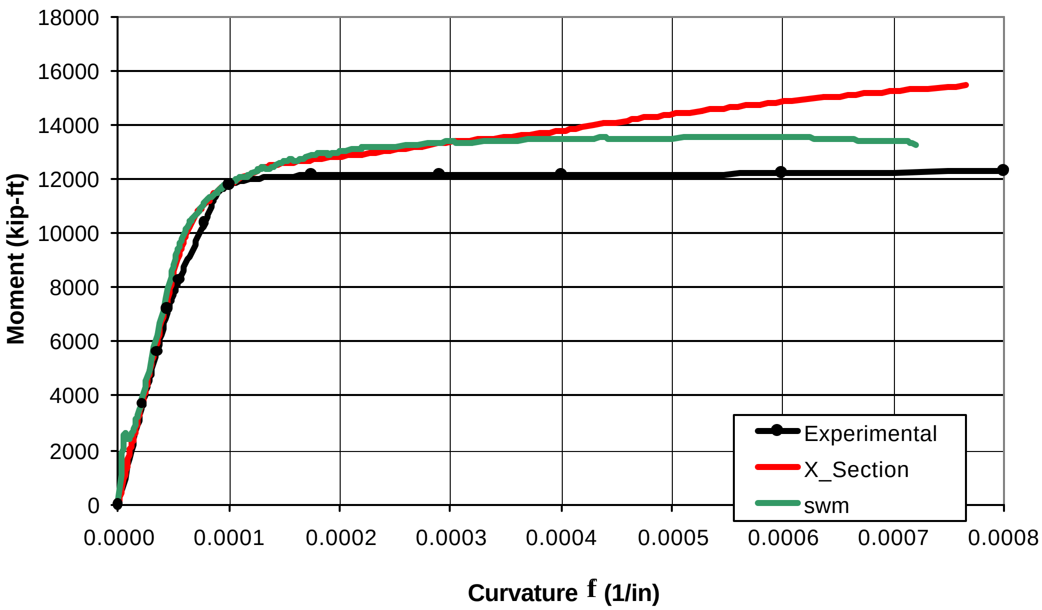

Fig. 8-16 Moment-Curvature Relationship of the 6-ft-Diamter Cross-Section

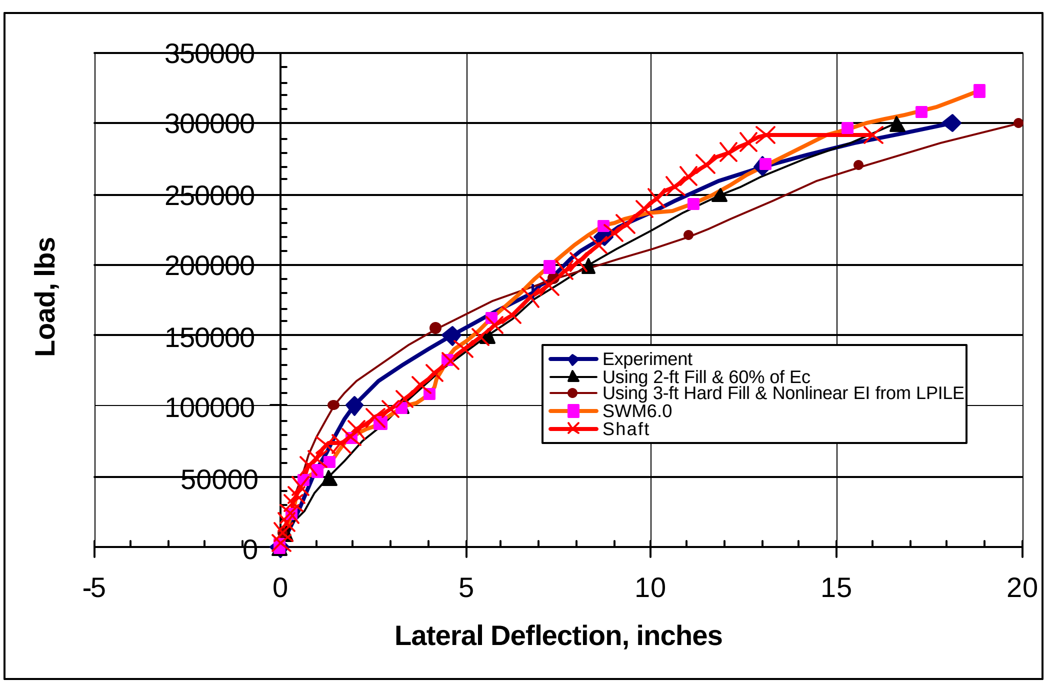

Fig. 8-17 Comparison Between Measured and Computed Lateral Deflection of the shaft tested at UCLA

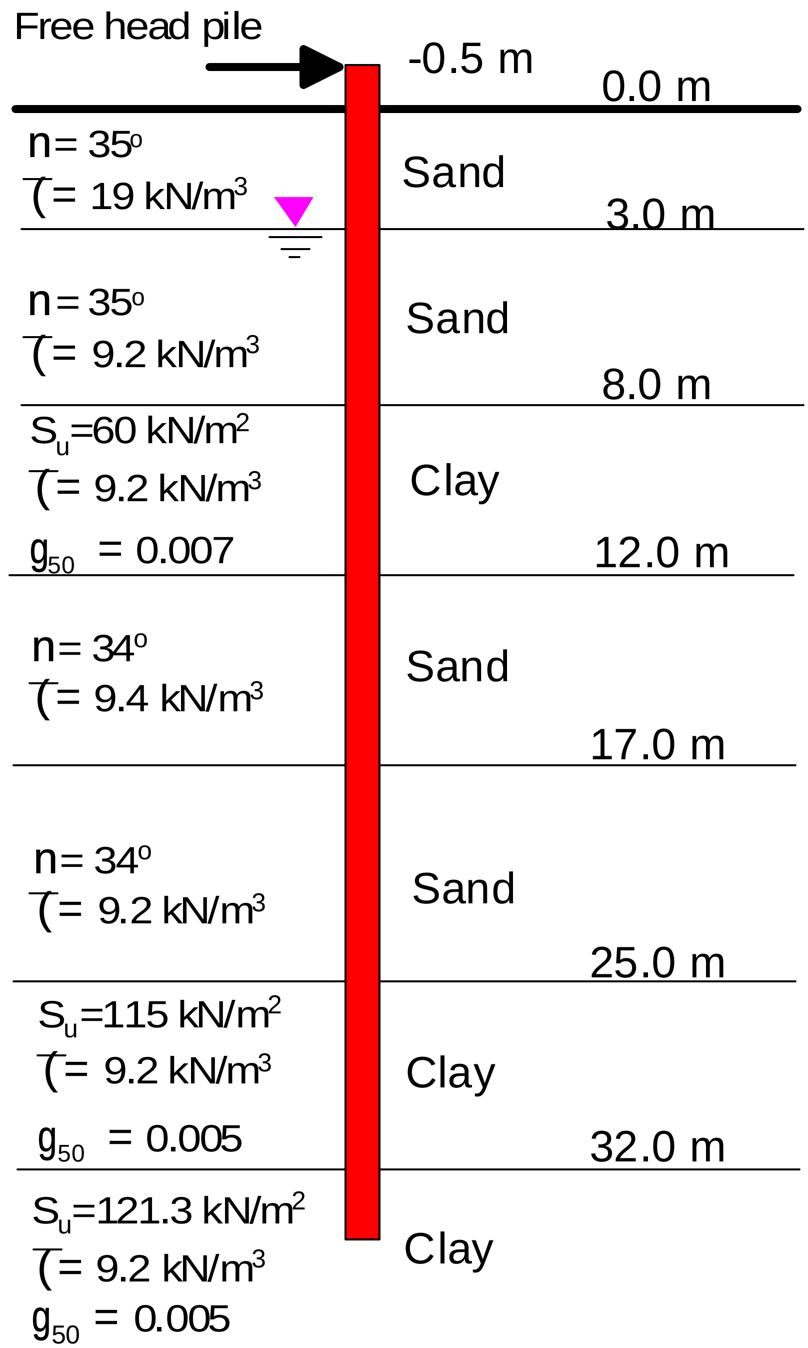

Fig. 8-18 Original Soil Profile and Pile Tested at the Chaiyi Test, Taiwan (Brown et al. 2001)

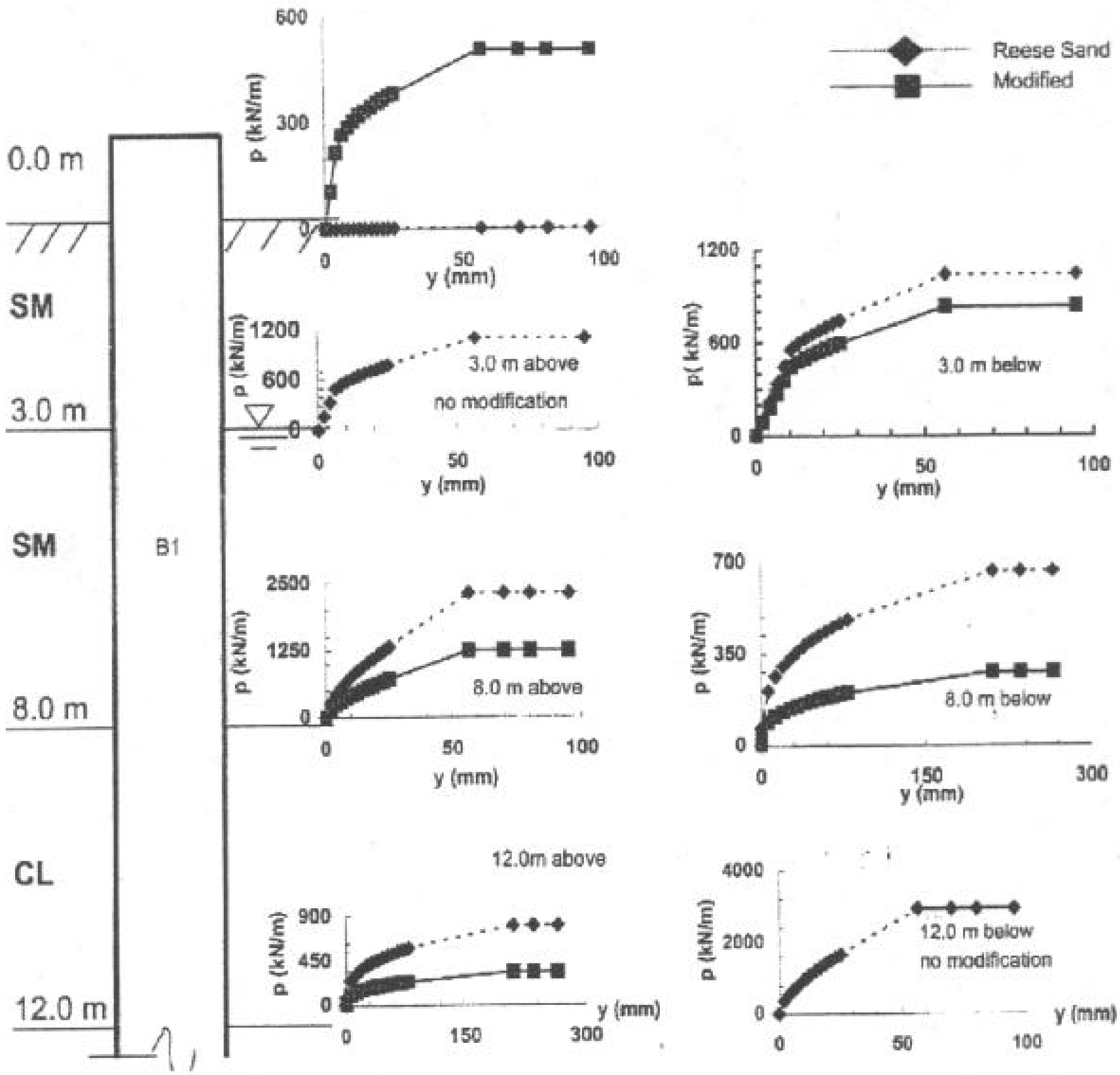

Fig. 8-19 Traditional p-y Curves Modified to Obtain Good Match with Field Data (Chaiyi Test, Brown et al. 2001)

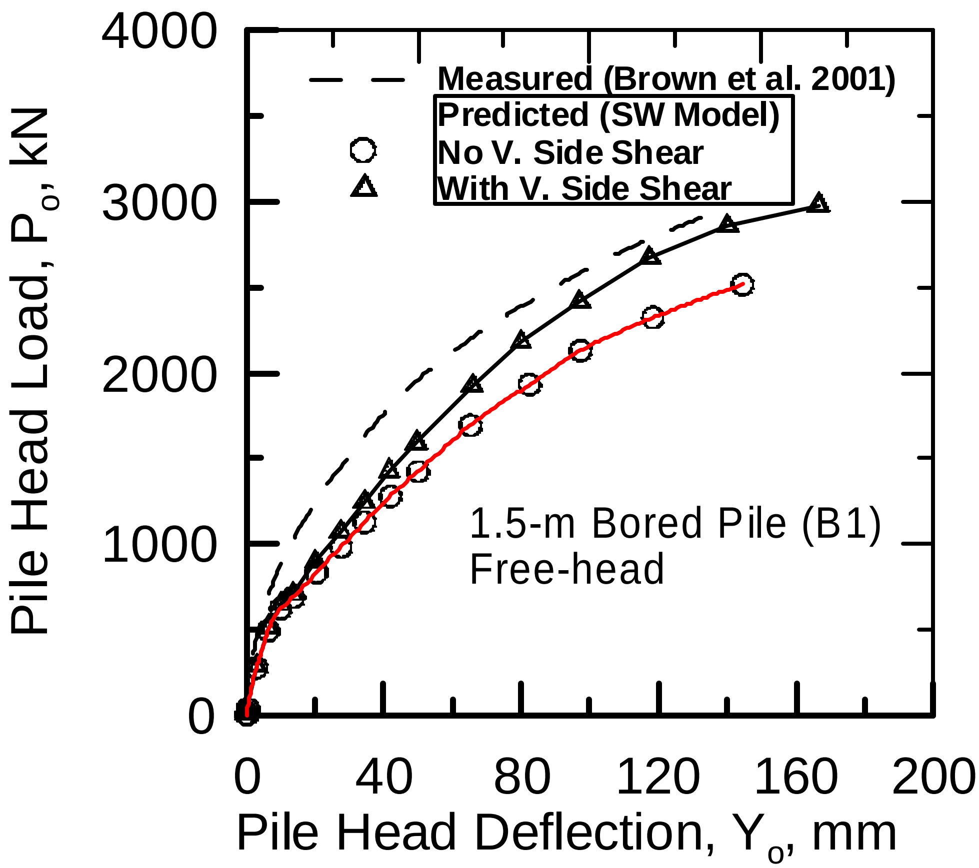

Fig. 8-20 Measured Vs. Computed Pile-Head Deflections and the Effect of Vertical Side Shear